684

VOLUME 81 SEPARATE No. 684

PROCEEDINGS

AMERICAN SOCIETY
OF
CIVIL ENGINEERS

MAY, 1955

ECONOMY AND SAFETY OF DIFFERENT TYPES OF CONCRETE DAMS

by August E. Komendant

STRUCTURAL DIVISION

{Discussion open until September 1, 1955}

Copyright 1955 by the AMERICAN SOCIETY OF CIVIL ENGINEERS
Printed in the United States of America

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

THIS PAPER

--represents an effort by the Society to deliver technical data direct from the author to the reader with the greatest possible speed. To this end, it has had none of the usual editing required in more formal publication procedures.

Readers are invited to submit discussion applying to current papers. For this paper the final date on which a discussion should reach the Manager of Technical Publications appears on the front cover.

Those who are planning papers or discussions for "Proceedings" will expedite Division and Committee action measurably by first studying "Publication Procedure for Technical Papers" (Proceedings – Separate No. 290). For free copies of this Separate—describing style, content, and format—address the Manager, Technical Publications, ASCE.

Reprints from this publication may be made on condition that the full title of paper, name of author, page reference, and date of publication by the Society are given.

The Society is not responsible for any statement made or opinion expressed in its publications.

This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N. Y.

ECONOMY AND SAFETY OF DIFFERENT TYPES OF CONCRETE DAMS

August E. Komendant[1]

INTRODUCTION

Of various types of dams the gravity dams, the arched gravity dams, the arch dams, and the buttressed dams combined with multiple arches, slabs, or domes are current practice. In the United States, the greatest dam builder in the world, for narrow and deep valleys gravity dams or arched gravity dams are used, and for relatively flat valleys earth-fill dams or gravity dams are the standard. In other countries, especially in Europe, for damming off narrow and deep valleys arch dams are utilized, and for flat valleys buttressed dams are preferred. In most recent designs the application of prestressing is favored for buttresses.

Which dam design should be advocated, with respect to its economy and safety, for a certain location has been the subject of many scientific works and comparative studies made during the last decades. It is apparent that the geology and topography at the dam site determine to a certain degree the general features of the design, but for typical conditions the economy of a design is mainly controlled by the following factors: quantities of material and labor required for construction, their price and relationship; time necessary for erection of the dam, and maintenance costs. On the other hand, the quantities required are dependent upon how efficiently the materials are used and upon the safety required. The labor and time required for erection are to a great degree controlled by the quantities, dimensions, and methods of placement of the concrete. The maintenance costs are dependent on the type of the dam and quality of concrete. To throw light on the effect of the above mentioned factors, upon the economy and safety of a design, different concrete dam types will be discussed below.

Gravity Dams

The controlling principle for design of a gravity dam is that no tensile stresses should develop neither in the base nor in any other section of the dam. For assumed linear pressure distribution this condition can be satisfied only if the resultant of all the forces which act above the base or sections considered comes within the middle third of the base or section of the dam. Disregarding uplift, earthquake, and ice or wave pressure, the minimum width of the base required is given by the following equation

$$W \frac{b}{3} = P \frac{h}{3} \qquad (1)$$

1. Doctor of Eng., Cons. Eng., New York, New York.

wherein W is the total weight of the unit length of the dam having the height h. P is the total water pressure acting upon the unit length. Assuming a triangularly shaped cross section, the weight of the dam is

$$W = \frac{hb}{2} \gamma_c$$

The total water pressure is

$$P = \frac{h^2}{2} \gamma_w$$

Substituting these values in Eq. (1) we obtain

$$b_{min} = h \sqrt{\frac{\gamma_w}{\gamma_c}} \qquad (2)$$

As the unit weight of water is γ_w = 64 lb per cu ft and the unit weight of concrete is γ_c = 150 lb per cu ft the minimum width of the base required is

$$b_{min} \cong 0.65\ h$$

However, considering the uplift, ice or wave pressure, and earthquake, the base width commonly required is up to b = 0.85 h. Regardless of the tremendously large width of the base and volume of concrete in the dam the safety of gravity dams is relatively small. The low safety is caused by the facts that linear stress distribution does not exist and extremely high stresses are developed by shrinkage and temperature stresses in such a massive concrete dam.

The variations in the elasticity of the foundation result in relatively high tensile stresses at the heel when the basin is filled and at the toe when the basin is empty. Such high tensile stresses have been measured at large scale models and can be easily proved also by means of Airy's stress function. Due to the high tensile stresses at the heel the full uplift develops resulting in reduced safety and favoring leakage of the gravity dam. Furthermore, it is a proved fact that in ordinary concrete works, especially when large volumes are involved, complete hydration of cement will never take place inside the concrete. Thus the variations in the elasticity and strength of the concrete can be very different throughout the dam. The reason for incomplete hydration can be explained as follows:

Cement consists of clinker fragments formed principally of

dicalcium silicate	$2CaOSiO_2$
tricalcium silicate	$3CaOSiO_2$
tricalcium aluminate	$3CaOAl_2O_3$
tetracalcium alumino-ferrite	$4CaOAl_2O_3Fe_2O_3$

The size of these fragments ranges approximately from 75 microns down to less than 1 micron. When water is added to these fragments, it is attracted by the grains resulting in softening the surface and forming calcium hydroaluminate and calcium hydrosilicate colloids. The speed of the water attraction and softening of the fragments is very different, it seems to be especially slow for

dicalcium silicate and fastest for tricalcium aluminate. By the chemical reaction heat is generated which creates vapor pressure etc., and hinders the rapid penetration of the grains of a larger diameter by water. Further, swelling of colloids occurs and the colloids are pressed together if the amount of water or space present is insufficient to allow spreading. As the result, clusters of slow softening fragments are formed, which further hinder the complete hydration of the cement grains and render these into colloidal solution in which state the crystallization takes place. The cluster formation is reduced if some admixture such as plastiment, air entrainment agents, etc., are added to the cement. Crystallization starts from the outmost surface of the grains or clusters where the hydration is more complete. In this state the capillary and crystallization pressure in concrete is very high. These pressures resist the swelling connected with hydration, thus further hydration of the nucleus of the fragments or clusters is hindered and delayed. It is possible that after years the hydration of cement grains inside of massive concrete is complete in no more than 1 to 2 microns from the surface of particles. The inside portion of particles larger than 4 microns in diameter is believed to be entirely in amorphous state. The amorphous substance offers relatively small resistance to the high capillary pressure and thus large scale shrinkage takes place in such a concrete. The magnitude of the capillary pressure present at the equilibrium of evaporation can be estimated by Freyssinet theory. In accordance with this theory the normal pressure in the capillary tubes is

$$p_n = \frac{2\gamma}{d_{\epsilon A}} \text{ mg per mm}^2 \qquad (3)$$

wherein γ is the capillary constant (approx. γ = 8 mg per mm), and $d_{\epsilon A}$ is the diameter of capillary tubes down to which size the complete evaporation of the water takes place under a given air humidity (ϵA). The value of $d_{\epsilon A}$ is

$$d_{\epsilon A} = \frac{2\gamma}{1.3 \times 10^7 \ln(1/\epsilon A)} \text{ mm} \qquad (4)$$

Assuming a moderate climate with relative humidity ϵA = 40 per cent, the pressure, p_n, is computed 17,000 psi. Estimating that the amount of tubes, up to the limiting size, is 4 per cent, the pressure will be approximately 700 psi. It is quite obvious that such a high, unequally distributed pressure over the cross section results in high tensile and shearing stresses in concrete. As the capillary stresses develop, the plastic flow of concrete takes place simultaneously; however, with a much lower rate than the stresses. This is especially true in the later state of the hardening of the concrete. Thus crack formation can be avoided or at least curbed if the development of capillary forces is delayed so that there is time for plastic flow to take place and for concrete to gain strength. To delay shrinkage and also to allow water to penetrate the cement particles, hydration heat must be cooled down, especially if placing of concrete in large quantities is required (up to 20,000 cu yard per day). This is done by means of artificial cooling (water cooling, ice lenses, etc.) of concrete and, recently, by reducing the cement amount in concrete and by using low heat cements. Regardless of those measures the high initial stresses cannot be avoided. This can be explained by the following: artificial cooling creates disuniformity of temperature distribution and

variation in elastic properties of concrete which leads again to increased high internal stresses resulting in increased crack formation. This is especially true if water cooling is used.

As the test specimens cut out of massive concrete show, the compressive strength of such a concrete can be relatively high which, of course, indicates only that the hydrated portion of the cement is strong enough to prevent lateral flow of particles but not enough to resist weathering and secure watertightness of concrete, requirements which are in dam construction far more important than the compressive strength. To control leakage through dam and corrosion of concrete, drainage and control gangs must be provided inside the dam.

The almost continuous change of pressure in the bedrock beneath the foundation favors the development of leakage. This all decreases the already low safety of gravity dams regardless of large masses involved and the very safe appearance of the dam. This fact is proved by the much easier destruction of gravity dams than arch dams in war.

The large masses, chemical heat, and shrinkage problems are the controlling factors for choice of construction methods, efficiency, and construction time of the dam. To satisfy even the minimum requirements for quality of the concrete in a dam there is up to date no other way than lengthening the construction time up to several years, utilizing special methods for cooling, placing of concrete in small sections, and providing joints. To avoid leakage the foundation for gravity dams must be very carefully constructed and very often even the bedrock density improved by forcing in large quantities of cement under high pressure. All this reduces the economy of gravity dams.

As to the amount of labor required for the construction of gravity dams, it is very often less than that for arch dams regardless of large volumes involved (see Arch Dams). This is obvious, because standard forms are used which do not require much labor. Furthermore, the simplicity of construction and large quantities justify almost entirely mechanized construction. As labor in the United States is relatively more expensive than material, which results in high mechanization of the large construction sites, the favoring of gravity dams, regardless of the many deficiencies inherent in the gravity type of dams, is well explained.

Arched Gravity Dams

The safety of an arched gravity dam is relatively higher than that of a gravity dam. Here the water pressure is believed to be partly transmitted by arch action into the flanks of the valley (see Arch Dams). The tensile stresses developed at the heel, resulting in increased uplift, do not reduce the safety of this dam type because the decreased cantilever action due to uplift will be automatically balanced by increased arch action. The other features of an arched gravity dam are the same as for gravity dams described in the previous section.

Arch Dams

The most economical arch dam would be a cylindrical arch in which the stresses are approximately equal across any section. This condition would be satisfied only in a dam with triangular cross section having zero wall thickness (t) at water elevation, a constant radius (r), and constant radial deflection throughout the dam. The arch thrust in such an arch is proportional to the depth z from water elevation and to the radius r of the arch. Thus

the normal force and corresponding normal stress are

$$N_{\beta o} = z\,\gamma_w\,r_e \quad \text{(cylinder formula, indicated by index o, } r_e \text{ - radius of extrados)}$$

$$\sigma_{\beta o} = \frac{N_{\beta o}}{t_z} = \text{const.} \tag{5}$$

and the radial deflection for constant modulus of elasticity E is

$$\omega_o = \frac{\sigma_{\beta o} r}{E} = \text{const.} \tag{6}$$

It is apparent that such an arch dam is impossible mainly because the radial deflection of the wall, ω, at its boundaries is approximately zero and the edge of the wall cannot be made movable. Furthermore, in case of large base width the wall is restrained in the foundation. As a consequence, the water pressure at the bottom part of the dam is vertically transmitted by cantilever action into the bedrock. This cantilever action is assumed to be limited to the lower third of the dam. However, this is not true and the cantilever action affects the stress distribution in the wall almost throughout the entire height of the dam. As the result of boundary disturbances, large flexural stresses are developed in arch dams which lead to extensive crack formation. To estimate the flexural stresses even in a reasonable degree makes the analysis of an arch dam rather complicated. Furthermore, the most advanced theories involve numerous assumptions and introduce uncertainties.[2]

To throw light on the safety and economy of the arch dams their stress condition will be considered in the following. However, since the complete stress analysis of dams is out of the scope of this paper, the stresses will here be discussed only to the extent necessary in the development of the subject.

Each section of the wall under consideration is subjected to normal water pressure $p = z\gamma_w$, z being the distance of the section from the water level. A part of this pressure p is transmitted by arch action (p_β) while the remaining part $p_z = p - p_\beta$ is transmitted by cantilever action (see Fig. 1). p_β develops the thrust in the line of pressure $H = p_\beta r$, r being the radius of the arch. The strain caused by the thrust (e), shrinkage (s), change of temperature ($T, \Delta T$), and displacement of abutments (Δ) results in a reduction

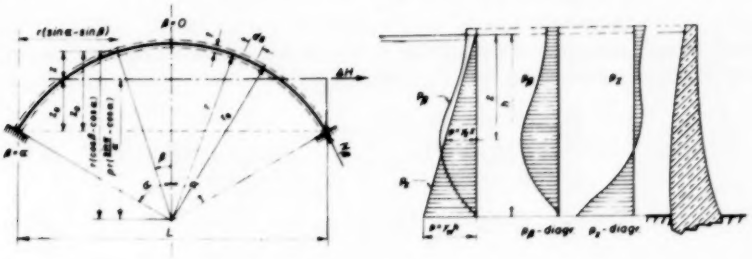

Fig. 1

2. See F. Toelke, Wasserkraftanlagen, Edited by A. Ludin, Springer Verlag, Berlin, 1938.

ΔH of the statically determinate thrust H. If the water pressure transmitted by arch action (p_β) is known, the magnitude of H can be computed for any restraint ratio ρ at the abutment applying the theory of elasticity. It is

$$\Delta H = - \frac{\delta_{ao}}{\delta_{aa}} = H_e + H_s + H_T + H_{\Delta T} + H_\Delta \tag{7}$$

wherein δ_{ao} is the change of span of the statically determinate circular arch due to the thrust caused by water pressure (p_β), shrinkage, uniform and unequal temperature change, and displacement of abutments, δ_{aa} is the change of the span due to $\Delta H' = 1$. They are:

$$\delta_{ao} = \delta_{ae} + \delta_{as} + \delta_{aT} + \delta_{a\Delta T} + \delta_{a\Delta}$$

$$= 2\int_{\beta=o}^{\alpha} N_\beta \cos\beta \frac{r d\beta}{EA} + \epsilon_s 2r\sin\alpha \mp \epsilon_T 2r\sin\alpha - 2\frac{\epsilon \Delta T}{t}\int_{\beta=o}^{\alpha} \left((\cos\beta - \cos\alpha) - \rho\left(\frac{\sin\alpha}{\alpha} - \cos\alpha\right)\right) r d\beta + 2\Delta \tag{8}$$

$$\delta_{aa} = 2\int_{\beta=o}^{\alpha} (z_o^2 - z_o z_e)\frac{ds}{EI} + 2\int_{\beta=o}^{\alpha} \cos^2\beta \frac{ds}{EA}$$

wherein ϵ_s is the change in length due to shrinkage, $\epsilon_T = \alpha T$ is the change in length due to temperature change T, $\epsilon_{\Delta T}$ is the lengthening of the fiber with respect to center line due to unequal temperature change and is considered to be positive if air side fiber is lengthened.

The ordinates are (Fig. 1):

$$z_o = r(\cos\beta - \cos\alpha)$$

$$z_e = \rho r\left(\frac{\sin\alpha}{\alpha} - \cos\alpha\right) \tag{9}$$

$$z = z_o - z_e$$

After integration, rearranging the expressions, and considering the influence of plastic flow upon the H_s and H_Δ we obtain:

$$H_e = - N_\beta C_H$$

$$H_s = - \epsilon_s E t C_H \frac{1 - e^{-\psi}}{\psi}$$

$$H_T = \mp \epsilon_T E t C_H \tag{10}$$

$$H_{\Delta T} = \epsilon_{\Delta T} E r C_H f_{\Delta H}$$

$$H_\Delta = - \frac{\Delta E t}{r\sin\alpha} C_H e^{-\psi}$$

wherein

$$\psi = \frac{\text{plastic flow}}{\text{elastic strain}} \sim 2.0$$

$$e = 2.71$$

$$f_{\Delta H} = (1 - \alpha\cot\alpha)(1 - \rho)$$

$$C_H = \frac{\sin\alpha}{\frac{1}{4}(\sin 2\alpha + 2\alpha) + 12\frac{r^2}{t^2}\left(\alpha(\cos^2\alpha + \frac{1}{2}) - \frac{3}{4}\sin 2\alpha - \rho\alpha(\frac{\sin\alpha}{\alpha} - \cos\alpha)^2\right)}$$

The C_H-value dependent on center angles and radius/depth ratios for an assumed ρ = 0.75 are represented in Fig. 2. The $f_{\Delta H}$ values for common center angles and for an assumed ρ = 0.75 are

α =	20°	25°	30°	45°	60°
$f_{\Delta H}$=	0.0102	0.0161	0.0233	0.0536	0.0989

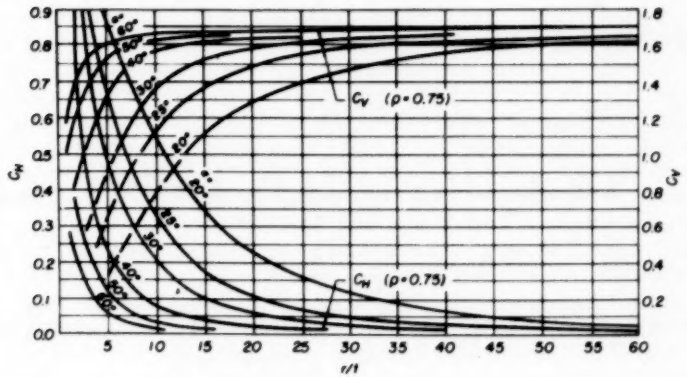

Fig. 2

In Fig. 2 C_H = values for estimating horizontal stress; C_V = values for estimating vertical stress; and center angle = 2α. The horizontal moments due to ΔH are

$$M_\beta = \Delta H\, r\left((\cos\beta - \cos\alpha) - \rho(\frac{\sin\alpha}{\alpha} - \cos\alpha)\right) \quad (11a)$$

At crown:
(β = 0)

$$M_{\beta c} = \Delta H\, r\left((1 - \cos\alpha) - \rho(\frac{\sin\alpha}{\alpha} - \cos\alpha)\right) \quad (11b)$$

at abutments:

$(\beta = \alpha)$

$$M_{\beta a} = -\Delta H\, r\, \rho\left(\frac{\sin\alpha}{\alpha} - \cos\alpha\right) \tag{11c}$$

and the corresponding stresses

at crown:

$$\sigma_{\beta c} = -\frac{H - \Delta H}{t} \pm \frac{6\Delta H}{t}\frac{r}{t}\left((1 - \cos\alpha) - \rho\left(\frac{\sin\alpha}{\alpha} - \cos\alpha\right)\right) \tag{12a}$$

at abutments:

$$\sigma_{\beta a} = -\frac{H - \Delta H}{t} \pm \frac{6\Delta H}{t}\frac{r}{t}\,\rho\left(\frac{\sin\alpha}{\alpha} - \cos\alpha\right) \tag{12b}$$

or in terms of normal stresses

$$\sigma_\beta = \frac{H - \Delta H}{t} \quad \text{and} \quad \Delta\sigma_\beta = \frac{\Delta H}{t}$$

$$\sigma_{\beta c} = -\sigma_\beta \pm 6\,\Delta\sigma_\beta \frac{r}{t}\left((1 - \cos\alpha) - \rho\left(\frac{\sin\alpha}{\alpha} - \cos\alpha\right)\right)$$

$$= -\sigma_\beta \pm \Delta\sigma_{\beta c} \tag{12c}$$

$$\sigma_{\beta a} = -\sigma_\beta \pm 6\,\Delta\sigma_\beta \frac{r}{t}\,\rho\left(\frac{\sin\alpha}{\alpha} - \cos\alpha\right)$$

$$= -\sigma_\beta \pm \Delta\sigma_{\beta a}$$

The values in parenthesis for crown and abutment are approximately equal for $\rho = 0.75$ and as average they are for $\alpha = 30^\circ$, 0.067 and for $\alpha = 60^\circ$, 0.25. Assuming the normal stress $\sigma_\beta = 1000$ psi, shrinkage $= 35 \times 10^{-5}$, plastic flow $\psi = 2.0$, change of temperature in concrete $\epsilon_T = -20^\circ$ F $\times$ 6.0×10^{-6}, and modulus of elasticity $E = 3.0 \times 10^6$ we obtain the normal and flexural stresses for the ratio $r/t = 4$ and $r/t = 35$ for $\alpha = 30^\circ$ and $\alpha = 60^\circ$ by Eqs. (10) and (12c)

$\alpha = 30^\circ$

$r/t = 4$: $\sigma_\beta \pm \Delta\sigma_\beta = 0 \pm 1685$ psi

$r/t = 35$: $\sigma_\beta \pm \Delta\sigma_\beta = -962 \pm 440$ psi

and

$\alpha = 60^\circ$

$r/t = 4$: $\sigma_\beta \pm \Delta\sigma_\beta = -857 \pm 856$ psi

$r/t = 35$: $\sigma_\beta \pm \Delta\sigma_\beta = -998 \pm 105$ psi

This illustration indicates clearly that the flexural stresses increase as the ratio r/t and α decrease. As a consequence, the deeper the considered arch ring is located the more the flexural stresses increase. For small ratio r/t and center angle α the direct compressive stress (arch action) is negligible in comparison with flexural stresses (beam action) and thus in such cases the water pressure p_β is transmitted to the flanks of the valley almost entirely by beam action. In case of constant angle dams (JORGENSEN Dam) the decrease of the ratio r/t at the lower part of the dam is counteracted by a large center angle α. This fact makes the constant angle dam far superior over the conventional type of arch dams. But even for moderately large center angle and fairly large ratios r/t the combined flexural stresses Eqs. (10) become critical and overcome the tensile strength of the concrete. As a result, extensive crack formation appears in the tension zones of the arch dams especially of the earlier designs having relatively large base width and small center angles.

As to the water pressure distribution between the arch (p_β) and the cantilever (p_z) action, there is no simple method for their determination. It can be done by considering the dam divided into vertical and horizontal strips. Each such strip is loaded with water pressure p_z and p_β respectively. For the cross points the deflection of strips must be equal due to the continuity. This method involves a large number of simultaneous linear equations which have to be solved for p_z or p_β. To obtain the desired accuracy the number of equations required becomes rather high and their solution complicated and time requiring. Furthermore, this method does not take into account the influence of the twisting moments and thus is a rough approximation. The same applies for the trial load method. The best known and simplest solution of the problem is the method of successive approximation. In accordance with this method the dam is considered as a cylindrical shell with varying thickness. The stresses in such a shell are determined by GECKELER's Method.[3] The dam wall is divided into sections wherein each section is considered as a cylindrical shell with assumed constant depth. Starting from the bottom edge the boundary forces acting at the top edge of the first shell serve as the boundary forces for the bottom edge of the following section etc. This way the internal forces N_β, Q_z, and M_z or respective stresses and displacements in horizontal and vertical direction can be determined. The water pressure in both directions can be obtained by differentiation. However, these stresses and the water pressure distribution are not the true ones, because the dam does not act as a closed cylinder but as a segment of the cylinder, i.e., the points at abutments are not free to move in radial direction as it is the case with closed cylinder. The effect is similar to the displacement of the abutments and results in increased deflection and corresponding higher vertical stresses in the dam. The actual stresses and water pressure distribution can be obtained by successive approximation from the results of the arch action

3. J. W. Geckeler, Ueber die Festigkeit achsensymmetrischer Schalen, Forsch.-Arb. Ing.-Wes., Berlin, 1926.

(Eqs. (7) through (12)) and values from the closed cylindrical shell theory. The water pressure distribution for a trapezoidal cross section is illustrated in Fig. 1.

Results obtained by application of the method of successive approximation reveal that the cantilever influence at the bottom edge of the variable cross section is slightly higher than for cylindrical shell having constant thickness. However, the damping of the moments upwards is greater because the characteristic length L becomes smaller with decreasing thickness of the wall. The vertical stress for constant thickness cylindrical shell is

$$\sigma_z = 1.74\ (\sigma_{\beta o} + L\ \frac{d\sigma_{\beta o}}{dz}) \qquad (13a)$$

Disregarding the relatively small value of $d\sigma_{\beta o}/dz$ for the bottom edge of the wall the maximum vertical stress due to the water pressure becomes:

$$\sigma_z = 1.74\ \sigma_{\beta o} \qquad (13b)$$

wherein $\sigma_{\beta o} = N_{\beta o}/t = r_e p/t$ is the cylindrical stress at the bottom edge of the wall.

The stress given by Eq. (13) is required to balance the radial deflection of the wall, w_o, due to the water pressure only. Considering the influence of the horizontal moments, temperature changes, shrinkage, and change in the span length of the wall the maximum vertical stress (σ_z) developed at the edge becomes C_V times (see Fig. 2) higher than the stress given by Eq. (13b) (see further Leaf Spring Type Shell Dam).

Following from the above discussion the flexural stresses in arch dams are very large. As the result, numerous cracks are formed in the tension zones both in vertical and horizontal directions of the arch. However, regardless of the extensive cracks found, not a single arch dam has failed for structural weakness. This is explained by the following facts:

The crack development in an arch dam results in a formation of a relatively thin secondary arch in which water pressure is not any more transmitted by flexural stresses (beam action) but mainly by normal stresses (σ_β) to the flanks and bottom of the valley. Thus the loss in cross section because of cracks is balanced by more efficient water pressure transmission to the foundations. The structural safety of arch dams seems not to be reduced by crack formation but rather to be increased. However, the safety may be considerably decreased if corrosion due to the crack formation is allowed to develop.

The above discussion shows that the efficiency and safety of an arch dam increases with the decrease of the thickness and radius of the wall. Making use of this fact and to increase the economy of the arch dams the most recent French dams have been designed relatively thin. The base width has been reduced down to b = 0.17 h or approximately 1/4 of the minimum base width required for gravity dams. By this way the volume of concrete could be reduced down to 25 per cent of that of gravity dams (see Vertical Stresses in following section).

In the Jorgensen dam the radius decreases constantly downwards, thus the flexural stresses are here reduced because of small radius and large constant center angle. Therefore, the material in this design is more efficiently used than in the other types of arch dams. But even this design is far from

ideal conditions where the entire water pressure is transmitted by normal stresses.

As the mass of arch dams varies from 25 to 40 per cent of that of gravity dams, the use of material in arch dams is far more economical than in gravity dams. However, their economy is greatly reduced because the extrados and intrados of slender arch dams are in two ways curved which makes the form-work complicated and expensive, especially in countries with a high living standard. This is one of the reasons why the gravity dams, the construction of which is simple and for which standard forms can be used, are favored in the United States. The second reason seems to be the complexities of the design of the arch dams.

Leaf Spring Type Shell Dam

By this dam type the aims: simplicity in design and construction, safety, and economy are realized. The water pressure is almost entirely transmitted by arch action into the flanks of the valley. Always higher compressive stresses in bedrock at the water side than at the air side of the dam, regardless of whether the basin is empty or filled, which is not possible in any other known type of dam, results in higher safety for this dam type. Erection of the cylindrical shells can be accomplished by use of slip forms. In principle the design is illustrated in Fig. 3.[4] In Fig. 3: 1. Prestressed Abutments; 2. Asphalt Layers; 3. Hinges; 4. Asphalt; and 5. Foundation. The shell arches with decreasing heights are numbered 1 to n (n = 3 in Fig. 3) each shell sustains approximately the same hydrostatic pressure the height of which, however, varies. The shells are separated with thin layers of

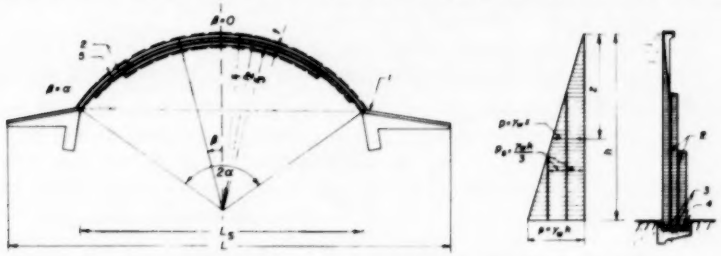

Fig. 3. Leaf Spring Type Shell Dam Plan and Sections

asphalt or equal material of adequate consistency. By choice of a convenient consistency it is possible to eliminate the sliding friction of the separating layer entirely, especially when considering that pressure upon the separation layer is continuous and with long endurance. The advantage of the asphalt layer is an increased watertightness of the dam.

The basic idea of such a dam type is not a new one. To the writer's knowledge as early as in 1927 MESNAGERS and VEYRIER (French engineers) proposed the use of five entirely separated very slender reinforced concrete arched dams, arranged behind each other with decreasing height separated with water space, instead of one thick arch dam for the Marège dam on the Dordogne River, in France. Each of the five arches sustains the same hydrostatic pressure. A similar stepped-off dam type has also been proposed by

4. This dam design was prepared and proposed by Dischinger and Komendant for Silvenstein Dam on the Isar River, Germany.

MAILLARD (Swiss engineer). However, the most complete type of such a dam has been developed by the American engineer F. A. NOETZLI.[5] The Noetzli's dam—Laminated Arch Dam with Forked Abutments—is characterized by the following features: The use of a special type of buttress to take the thrust of the upper arch elements, thereby reducing the required maximum arch span, and hence the maximum radius. The pronounced sloping, upstream from the crest of the crown of the dam, thereby securing the minimum radii for the lower arch elements. By the subdivision, where necessary, of the total arch thickness into two or more separate arches, thereby keeping the arches thin or relatively so, and maintaining throughout the dam the corresponding advantages.

Regardless of the technical superiority of these three earlier proposals above gravity and conventional arch dams, they have failed to draw sufficient attention and prove their superiority for utilization. The reasons for that are believed to be the following:

The separated structures of the French and Swiss engineers require notably more form work and their foundations are more expensive than for a single structure. Furthermore, if any one of the separated structures fails (war events, leakage, etc.) the entire dam will collapse by chain reaction. As to the Noetzli's dam, the strongly pronounced crest and two-way curved extrados and intrados of each lamina make the form work complicated and require much labor. The chemical heat problems will be similar as for any other massive dam. Due to the overhang it is difficult to avoid tensile stresses at the heel of the dam. The use of forked abutments introduces a point for differential displacements resulting in increased danger of crack formation. In the leaf spring shell dam design all these objections are eliminated as will be proved in the following discussion.

The safety of the leaf spring shell dam is mainly controlled by the stability against buckling. In accordance with Euler's formula the buckling strength of a straight column is

$$P_B = \frac{\pi^2 E I}{L_B^2} \tag{14}$$

or when introducing $I = Ai^2$ and taking $i^2 = t^2/12$ the buckling stress will be

$$\sigma_B = \frac{\pi^2 E\, t^2}{12\, L_B^2} \tag{15}$$

For a restrained cylindrical shell, having center angle 2α, the buckling length (L_B) is

$$L_B = \frac{1}{3}\, 2\alpha\, r \tag{16}$$

Substituting this value for L_B in formula Eq. (15) and assuming the buckling stress $\sigma_B = n\,\sigma_d$, wherein n is the factor of safety against buckling and

5. F. A. Noetzli, Laminated Arch Dams with Forked Abutments, Proc. Am. Soc. of Civil Engineers, S. 261-292, 1930.

σ_d is the allowable design stress, we obtain the limiting ratios r/t for individual shells

$$\frac{r}{t} = \frac{1.364}{\alpha}\sqrt{\frac{E}{n\sigma_d}} \tag{17a}$$

or taking n = 4

$$\frac{r}{t} = \frac{0.68}{\alpha}\sqrt{\frac{E}{\sigma_d}} \tag{17b}$$

For commonly encountered center angles α (α = 50° to 60° for the upper part of the shells) and lowest value (4000) of E/σ_d the ratio r/t must lie approximately between 40 and 50 to avoid buckling. The minimum thickness for the individual shells, as controlled by buckling, is

$$t_{min} = \frac{r\,\alpha}{0.68\sqrt{\frac{E}{\sigma_d}}} \sim \frac{r}{40 \text{ to } 50} \tag{18}$$

The horizontal stresses in the shells can be easily estimated by the Eqs. (10), (11), and (12). As shown previously, for the large center angles α and large ratios r/t the horizontal flexural stresses are negligible in comparison with normal stresses. For small center angles, i.e., at the lower portion of the dam, the N_β or σ_β values are superposed by boundary forces which results, respectively, in reduced horizontal flexural stresses. As a consequence, the horizontal flexural stresses are of secondary order. Considering that they are superposed by high normal stresses, no crack formation occurs in horizontal direction in the shell dam.

The radial deflection of the determinate shells due to the horizontal forces $N_{\beta o} + \Delta H$ and moment M_β are

$$\omega_o = \frac{1}{EA}\int_{\beta=o}^{\alpha}(N_{\beta o} + \Delta H\cos\beta)\sin\beta\, r\,d\beta + \frac{\Delta H r^3}{EI}\int_{\beta=o}^{\alpha}\left((\cos\beta - \cos\alpha) - \rho\left(\frac{\sin\alpha}{\alpha} - \cos\alpha\right)\right)(\sin\alpha - \sin\beta)\,d\beta \tag{19a}$$

Substituting $I = t^3/12$ and $A = t$ after integration we obtain

$$E\omega_o = N_{\beta o}\frac{r}{t}\left((1 - \cos\alpha) - \frac{\Delta H}{4N_{\beta o}}(1 - \cos 2\alpha)\right) + 12\Delta H\left(\frac{r}{t}\right)^2 r\, f_\omega$$

$$\omega_o = \frac{N_{\beta o}}{E}\frac{r}{t}\Big((1 - \cos\alpha) + \frac{\Delta H}{N_{\beta o}}\Big(12\Big(\frac{r}{t}\Big)^2 f_\omega - \frac{1}{4}(1 - \cos 2\alpha)\Big)\Big) = \frac{N_{\beta o}}{E}\frac{r}{t}\,C_V \tag{19b}$$

The radial deflection in terms of normal stress

$$\sigma_{\beta o} = \frac{N_{\beta o}}{t}\,,\quad \Delta\sigma_\beta = \frac{\Delta H}{t}$$

is

$$\omega_o = \frac{\sigma_{\beta o}}{E}\,r\Big((1 - \cos\alpha) + \frac{\Delta\sigma_\beta}{\sigma_{\beta o}}\Big(12\Big(\frac{r}{t}\Big)^2 f_\omega - \frac{1}{4}(1 - \cos 2\alpha)\Big)\Big) = \frac{\sigma_{\beta o}}{E}\,r\,C_V \tag{19c}$$

wherein $\sigma_{\beta o}$ is the normal stress due to the water pressure ($p_s = p/n$) $\Delta\sigma_\beta$ the change of the normal stress ($\sigma_{\beta o}$) due to the rib shortening, shrinkage, temperature change, etc. and

$$f_\omega = \sin^2\alpha - \frac{\alpha}{2}\sin 2\alpha - \frac{1}{4}(1 - \cos 2\alpha) + (1 - \cos\alpha)\cos\alpha - \rho\Big(\frac{\sin\alpha}{\alpha} - \cos\alpha\Big)(\alpha\sin\alpha + \cos\alpha - 1) \tag{19d}$$

The f_ω-values for common center angles are given below ($\rho = 1.00$)

α =	25°	30°	35°	40°	45°	50°	60°
f_ω =	0.00145	0.00293	0.00532	0.01141	0.0137	0.01757	0.0384

The first part in the parenthesis (Eqs. (19b) and (19c)) indicates the deflection due to the determinate cylindrical stress($\sigma_{\beta o}$) and the second part the influence of horizontal bending and the rib shortening due to the indeterminate stress caused by the arch thrust (ΔH). The value in parenthesis (C_V) is almost constant for large center angle α and seems to be not much affected by the ratio r/t, it can be commonly up to 1.7 (see Fig. 2). However, for smaller center angles it is much affected by r/t and can become even <1.

The radial deflection ω_o by Eq. (19c) and angular rotation $d\omega_o/dz$ must be zero at the bottom edge of the shell. These conditions can be satisfied by introducing a horizontal boundary force X_1 and a boundary moment X_2 at the bottom edge of the shell to balance ω_o and $d\omega_o/dz$. In accordance with the theory of cylindrical shells[6] the values of X_1 and X_2 are

6. K. Beyer, Die Statik in Eisenbetonbau, Band 2, Springer Verlag, Berlin, 1934.
A. E. Komendant, Prestressed Concrete Structures, McGraw-Hill Book Co., Inc., New York, 1952.

$$X_1 = \frac{LE}{2\frac{r}{t}\,r}\left(2\omega_o + \frac{d\omega_o}{d\xi}\right) = \frac{0.38E}{\sqrt{\left(\frac{r}{t}\right)^3}}\left(2\omega_o + \frac{d\omega_o}{d\xi}\right)$$

$$X_2 = \frac{L^2}{2\frac{r}{t}\,r}E\left(\omega_o + \frac{d\omega_o}{d\xi}\right) = \frac{0.29E\,t}{\frac{r}{t}}\left(\omega_o + \frac{d\omega_o}{d\xi}\right) \qquad (20)$$

wherein

$$\xi = z/L$$

$$L = \frac{0.76r}{\sqrt{r/t}}$$

L is termed the characteristic length of cylindrical shell. The values of normal force N_β and moment M_z are obtained by superposition

$$N_\beta = N_{\beta o} + X_1 N_{\beta 1} + X_2 N_{\beta 2}$$

$$M_z = M_{zo} + X_1 M_{z1} + X_2 M_{z2} \qquad (21a)$$

wherein $N_{\beta 1}$, $N_{\beta 2}$ and M_{z1}, M_{z2} are the normal forces and moments in the statically determinate shell due to the boundary forces $X_1 = 1$ and $X_2 = 1$ respectively. They are

$$N_{\beta 1} = -\frac{2r}{L}\,e^{-\xi}\cos\xi$$

$$N_{\beta 2} = \frac{2r}{L^2}\,e^{-\xi}(\cos\xi - \sin\xi)$$

$$M_{z1} = -L\,e^{-\xi}\sin\xi \qquad (22a)$$

$$M_{z2} = e^{-\xi}(\sin\xi + \cos\xi)$$

Introducing the values from Eqs. (20) and (22a) into Eqs. (21a) we obtain

$$N_\beta = N_{\beta o} - \frac{E}{r/t}\,e^{-\xi}\left(\left(2\omega_o + \frac{d\omega_o}{d\xi}\right)\cos\xi + \left(\omega_o + \frac{d\omega_o}{d\xi}\right)(\cos\xi - \sin\xi)\right) \qquad (23)$$

$$M_z = -\frac{0.29Et}{r/t}\, e^{-\xi}\left((2\omega_o + \frac{d\omega_o}{d\xi})\sin\xi + (\omega_o + \frac{d\omega}{d\xi}\Omega)(\sin\xi + \cos\xi)\right)$$

The water pressure carried by cantilever and arch action can be obtained by differentiation of M_z. Thus

$$p_z = \frac{d^2M_z}{dz^2} = \frac{d^2M_z}{L^2d\xi^2} \qquad (24)$$

$$p_\beta = p_s - p_z$$

The actual deflection of the wall is obtained by applying the principle of superposition

$$\omega = \omega_o + X_1\omega_1 + X_2\omega_2 \qquad (21b)$$

wherein the ω_o is the deflection by Eq. (19c), ω_1 and ω_2 the deflection due to $X_1' = 1$ and $X_2' = 1$ respectively. They are

$$\omega_1 = \frac{2r}{L}\frac{r}{E}\frac{r}{t}\, e^{-\xi}\cos\xi$$

$$\omega_2 = -\frac{2r}{L^2}\frac{r}{E}\frac{r}{t}\, e^{-\xi}(\cos\xi - \sin\xi) \qquad (22b)$$

Introducing the ω_o value from Eq. (19c) into Eq. (20) and disregarding the value $d\omega_o/d\xi$ because it is relatively small for thin shells subjected to a constant pressure (p_s), we obtain the cantilever moment at the edge of the shell.

$$M_z = X_2 = 0.29t^2\sigma_{\beta o}\left((1 - \cos\alpha) + \frac{\Delta\sigma_\beta}{\sigma_{\beta o}}\left(12(\frac{r}{t})^2 f_\omega - \frac{1}{4}(1 - \cos 2\alpha)\right)\right) = 0.29t^2\sigma_{\beta o}C_V \qquad (25a)$$

and the corresponding vertical stress

$$\sigma_z = \frac{M_z}{S} = \frac{6M_z}{t^2} = 1.74\sigma_{\beta o}\left((1 - \cos\alpha) + \frac{\Delta\sigma_\beta}{\sigma_{\beta o}}\left(12(\frac{r}{t})^2 f_\omega - \frac{1}{4}(1 - \cos 2\alpha)\right)\right) = 1.74\sigma_{\beta o}C_V \qquad (25b)$$

Using the same values for computing $\Delta\sigma_\beta$ as on the page 8 the σ_z stresses for $\rho = 1.0$ are by Eq. (25b)

$\alpha = 30^\circ$ $r/t = 4$: $\sigma_z \cong 1.74 \times 0.57\ \sigma_{\beta o}$

$r/t = 35$: $\sigma_z \cong 1.74 \times 1.48\ \sigma_{\beta o}$

$\alpha = 60^\circ$ $r/t = 4$: $\sigma_z \cong 1.74 \times 1.50\ \sigma_{\beta o}$

$r/t = 35$: $\sigma_z \cong 1.74 \times 1.60\ \sigma_{\beta o}$

This example shows that the cantilever stresses cannot be reduced by lamination of the dam wall as was believed by Noetzli.[7] It is apparent that high stresses are developed in vertical direction of the wall, which cannot be taken by concrete and, as a result, large crack formation takes place at the bottom margin of the dam. The parenthesis values, $C_V < 1$, which appear with small r/t values indicate that arch action is negligible and the water pressure is mainly transmitted horizontally by beam action, i.e., that very high flexural stresses are developed at the lower part of the dam. In leaf spring shell dam the intolerable vertical stresses (σ_z) have been eliminated by providing hinge at the base which is easily possible because of the small thickness of the shells. Furthermore, such high stresses can also be greatly reduced by lowering the base somewhat below the elevation of impermeable stratum and thus decreasing the $N_{\beta o}$ value in such an extent that the σ_z becomes negligible.

The maximum vertical tensile stresses at the air side of the dam are approximately 40 per cent higher for hinged base than for fixed base. However, these stresses can be reduced by arranging the hinge eccentrically with respect to the center line of the wall.

The magnitude of σ_β and σ_z stresses with the hinged support can be easily computed in accordance with the Eqs. (19c) and (20). Considering that $X_2 = 0$ we obtain

$$N_\beta = N_{\beta o} - \frac{E}{r/t}\, e^{-\xi}\left(2\omega_o + \frac{d\omega_o}{d\xi}\right)\cos\xi$$

$$M_z = -\frac{0.29Et}{r/t}\, e^{-\xi}\left(2\omega_o + \frac{d\omega_o}{d\xi}\right)\sin\xi \qquad (26)$$

and

$$\sigma_z = \pm 1.74\sigma_{\beta o} e^{-\xi}\sin\xi\left((1 - \cos\alpha) + \frac{\Delta\sigma_\beta}{\sigma_{\beta o}}\left(12\left(\frac{r}{t}\right)^2 f_\omega - \frac{1}{4}(1 - \cos 2\alpha)\right)\right) = \pm 1.74\sigma_{\beta o} C_V e^{-\xi}\sin\xi \qquad (27)$$

The maximum value of $e^{-\xi}\sin\xi$ is approximately 0.32 and the value in parenthesis up to 1.7, the maximum vertical stress will be approximately in the same order as the cylindrical stress. As already stated, this stress can be

7. See page 12.

reduced by an eccentrical hinge. Thus the moment due to the eccentricity

$$M'_z = Wa$$

and the stresses in accordance with Eq. (27)

$$\sigma_z = \mp \frac{6Wa}{t^2} \pm 1.74\sigma_{\beta o} C_V e^{-\xi} \sin\xi \qquad (28)$$

wherein W is the weight of the wall above the section considered and a is the eccentricity of the hinge. By this way the σ_z-stress condition in the wall can be improved considerably.

Considering that the vertical stress is superposed by the normal stresses, due to the weight of the wall, and the tensile stresses appear at air side, no harmful crack formation is expected. However, if any doubt exists, the tensile stresses can be easily avoided by application of prestressing (see Buttressed Dams, Fig. 4).

The stress analysis of the leaf spring shell dam design is relatively simple in comparison with the conventional arch dam. Furthermore, in the theory of this dam no uncertain assumptions are required, but the stresses determined are of sufficient accuracy. Taking these facts into consideration, much higher allowable stresses can be used for the design without sacrificing safety.

The safety of the leaf spring shell dam against earthquake or other dynamic effects is considerably higher than of other types of dam because of the reduced masses and consequently small acceleration of forces acting upon the dam. Furthermore, the acceleration forces are greatly damped by the flexibility of the wall and the asphalt layer between the individual shells. Thus resonance possibilities are unlikely with shell dams.

The fact that the pressure upon the soil under foundation is independent of the filling height of the basin and it is always higher at the water side adds considerably to the safety against leakage through the base. Furthermore, the safety of the leaf spring shell dam is not affected by the uplift.

It is apparent that the allowable stresses are most efficiently used in leaf spring shell dam in comparison with other dams. The base thickness of gravity dams was approximately 0.85 h and as an average 0.25 h for arch dam but the base thickness of the shell dam can be less than 0.1 h. Thus the volume of concrete required would be approximately 15 per cent of that of gravity dams and 40 per cent of that of conventional arch dams. Making use of Noetzli's forked abutments for supporting the upper part of the water side shell the radius (r) and the span of the shells can be considerably reduced which allows the further reduction of the thickness of the shells and results in increased economy of the dam. Applying prestressing for abutments the displacements at the junction of the shell and abutment can be closely controlled and differential displacements entirely eliminated. The application of prestressing to the forked abutments not only reduces the possibility of crack formation but increases the economy and safety of the abutments and dam itself. In view of the construction time the use of forked abutments, which are to be considered as separated structures, can be completed simultaneously with the foundation of the dam. This simplifies the construction and reduces the construction time considerably.

In locations where the bedrock, upon which the dam is to be built, lies in great depth underneath the grade and the over-lying strata carry a ground water stream, the relatively small thickness of the leaf spring shell dam

makes it possible to erect the dam upon one single caisson or upon a row of caissons. Such a caisson can be easily constructed by use of slip forms. According to the process of dropping, the caisson is constantly increased in height above the ground water level. By this way the feasibility of such locations for damming off is made economically possible.

The constant and relatively small thickness of all shells allows the application of slip forms for pouring concrete. As an average it can be assumed that the wall is raised 8 in (20 cm) per hour or approximately 16 ft per 24 hours. Thus the placing of concrete of an individual shell, having a height of 600 ft, requires approximately 40 days (24 hour pour). The shell at water side of the dam will be first erected, then using the same forms the third shell is constructed, then the fifth, etc. The second and fourth shell require only a limited amount of forms because the adjoining shells provide the form for them. Because of the small thickness of shells and the possibility to execute the shells at separate times, the influence of chemical heat during hardening of the concrete is considerably reduced. Thus such a dam can be constructed in a relatively short time.

The depth of the slip form is about 4 ft and it can be used several times; approximately 40 hydraulic jacks can be lifted and controlled by only one man and the quantity of concrete required is small, it is obvious that the labor involved for the construction of the leaf spring shell dam is less than for any other dam type.

As to the quality of concrete poured by means of slip forms, there has been and still is a conception that concrete should not be disturbed in any way after the initial setting has started, i.e., after 1 hour, therefore, many engineers are skeptical about the quality of such a concrete. However, experience and tests show that the quality of concrete can be considerably increased when it is vibrated during the early stage of the initial setting. This can be explained by the following: During the period the cement grains become engaged with water and hydration of cement starts, great temperature differences and high capillary forces exist in concrete. By the vibration, due to the slip form and continuous placing of concrete, the temperature differences and capillary forces are more equally distributed in concrete which results in decreased initial stress and increased hydration of the cement grains. The formation of crystals in concrete is also increased by vibration. In accordance with the results of the tests made by the author, the best results have been obtained when vibration was applied approximately 3 to 4 hours after placing of the concrete. However, the time limit is largely controlled by the type of cement used, the amount of water present, the atmospheric temperature, and air humidity. Furthermore, the pour is continuous with slip forms, therefore, all construction joints and discontinuity places are eliminated which results in a very uniform concrete throughout the structure. The surfaces leaving the slip form (approx. after 5 hours) will be rubbed over which results in very dense concrete at the surface and in increased resistance against corrosion.

This method of placing the concrete makes it possible to start the filling of the basin much earlier than for other dams. After the first shell is constructed the filling can be started. Thus to the great economy of the construction cost a considerable saving of capital interests for the total project has to be added. As a conclusion of the above discussion, the savings in cost in comparison with the conventional arch dam is up to 50 per cent and even up to 70 per cent in locations favorable for this design. Without sacrificing safety, the construction time is reduced down to 1/3 of that required for the other dam types.

Buttressed Dams

The buttressed dams are used for damming off flat valleys. In such a location neither the conventional arch nor the leaf spring shell dams are suitable because of the large radius etc. (see previous section). The missing dead load of the dam to counteract the water pressure here must be balanced by water weight, i.e., the dam wall must be designed inclined in the direction to the air side. By this way the resultant of the forces acting upon the dam can be forced to remain within the middle third of the base. As the base for such a dam is mainly concentrated under the buttresses and is rather small, the uplift is not a serious problem. The safety of buttressed dams against overturning is relatively larger in comparison with gravity dams. However, rather small dimensions can lead to serious leakage and corrosion, especially if reinforcing is used.

As to the economy, it is obvious that the complicated and extensive formworks because of inclined elements are very expensive and time requiring, therefore, the savings in material will be consumed by higher labor costs. Due to this reason for moderate damming heights the gravity dams or earth dams are to be considered more economical than buttressed dams, especially in countries with a high living standard. However, using prestressed buttresses, the wall can be constructed vertical. Furthermore, if utilizing leaf spring shells or single shells between the buttresses, the number of intermediate buttresses can be reduced down to two and in this way even a relatively flat valley can be economically dammed off. The principle of such design is illustrated in Fig. 4. In Fig. 4: 1. Prestressed Buttresses; 2. Prestressed Abutments; 3. Prestressed Ribs; and 4. Prestressing Wire Ropes.

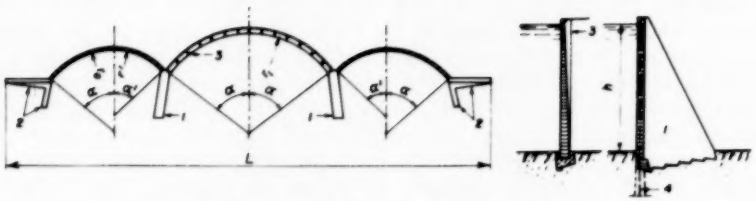

Fig. 4. Buttressed Shell Dam Plan and Sections.

The tensile stresses at the air side can be easily avoided here by providing ribs on the air side and prestressing the ribs. Thus, the shell acts in vertical direction as a T-beam. The cross section of the wall can be trapezoidal or of constant thickness. The buttresses will be prestressed by high strength wire ropes arranged as required in the ducts left for them by slip forms. After the prestressing the wire ropes will be grouted in. The wall as well as the buttresses are poured by means of slip forms.

This type of design is especially economical in locations where the stratum lying above the bedrock is relatively thick and impermeable or can be made impermeable economically, because here only the buttresses have to be founded on bedrock. Furthermore, this design can be used also in similar locations but where the stratum above the bedrock carries flowing ground water and has to be cut off by caisson. The very narrow width of the shell allows to construct the foundation much easier and more economically than for a thick wall. The general features and economy of such a dam are the same as for leaf spring shell dam described in previous section.

SUMMARY

The gravity dams, arched gravity dams, arch dams, leaf spring type shell dams, and buttressed dams have been critically considered and compared with respect to their safety and economy in this paper.

The criterion for safety of a dam type has been based upon the stress condition and corrosion. It has been shown by stress analysis that the stress condition of arch dams in vertical and horizontal directions is more severe than commonly realized. At the lower part even up to moderately large center angle and radius/depth ratio the water pressure is carried to the bottom and flanks of the valley mainly by beam actions. The arch action in this part of the wall develops after the crack formation has occurred. The most safe dams are the leaf spring shell and multiple single shell dams with prestressed buttresses. This conclusion is proved by shell theory; it is shown, that no tensile stresses occur in the shells and the stresses in soil beneath the foundation are always higher at the water side regardless of whether the basin is empty or filled, which is not possible in any other known type of dam.

The criterion for safety against corrosion is based upon the degree of hydration of the cement in massive concrete. The process and initial stresses during hydration, conditions favoring the hydration, and reduction of the initial stresses are described.

The criteria for economy have been based on the simplicity and exactness of the stress analysis, the efficiency of the use of materials, the simplest rapid construction possible, and on the lowest maintenance costs. It is proved that the leaf spring type shell and buttressed shell dams are also the dam types nearest to the ideal conditions. This conclusion is based on the following:

The design is simple and the stress conditions in shell can be described in desired limits without making uncertain assumptions.

The water pressure is transmitted to the flanks of the valley almost entirely by arch action resulting in the least possible volume of the dam without sacrificing the safety.

The shells are erected rapidly by slip forms lifted by hydraulic jacks in a continuous pour. One set of forms can be used for erection of all the individual shells. Chemical heat is not a serious problem because of the small thickness of the shells and the possibility to construct the individual shells in separate times.

The maintenance costs are low because of the high quality of concrete and dense surfaces obtained by slip forms, no crack formation and leakage.

AMERICAN SOCIETY OF CIVIL ENGINEERS

OFFICERS FOR 1955

PRESIDENT

WILLIAM ROY GLIDDEN

VICE-PRESIDENTS

Term expires October, 1955:
ENOCH R. NEEDLES
MASON G. LOCKWOOD

Term expires October, 1956:
FRANK L. WEAVER
LOUIS R. HOWSON

DIRECTORS

Term expires October, 1955:
CHARLES B. MOLINEAUX
MERCEL J. SHELTON
A. A. K. BOOTH
CARL G. PAULSEN
LLOYD D. KNAPP
GLENN W. HOLCOMB
FRANCIS M. DAWSON

Term expires October, 1956:
WILLIAM S. LaLONDE, JR.
OLIVER W. HARTWELL
THOMAS C. SHEDD
SAMUEL B. MORRIS
ERNEST W. CARLTON
RAYMOND F. DAWSON

Term expires October, 1957:
JEWELL M. GARRELTS
FREDERICK H. PAULSON
GEORGE S. RICHARDSON
DON M. CORBETT
GRAHAM P. WILLOUGHBY
LAWRENCE A. ELSENER

PAST-PRESIDENTS

Members of the Board

WALTER L. HUBER
DANIEL V. TERRELL

EXECUTIVE SECRETARY
WILLIAM H. WISELY

TREASURER
CHARLES E. TROUT

ASSISTANT SECRETARY
E. L. CHANDLER

ASSISTANT TREASURER
CARLTON S. PROCTOR

PROCEEDINGS OF THE SOCIETY

HAROLD T. LARSEN
Manager of Technical Publications

DEFOREST A. MATTESON, JR.
Editor of Technical Publications

PAUL A. PARISI
Assoc. Editor of Technical Publications

COMMITTEE ON PUBLICATIONS

SAMUEL B. MORRIS, *Chairman*

JEWELL M. GARRELTS, *Vice-Chairman*

GLENN W. HOLCOMB
OLIVER W. HARTWELL
ERNEST W. CARLTON
DON M. CORBETT